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DISCUSSION OF DESIGN OF PRESTRESSED TANKS

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DISCUSSION

L. J. Mensch,[20] M. ASCE.—Tanks and pipes are the most logical applications of prestressed concrete. As early as 1912 the city of Bologna, Italy, had a high-pressure prestressed pipe line built on the Siegwart system. Other early pipe lines have been reported from Switzerland and France.

Prestressed cast iron water pipes and guns have been in use since 1850 in France, England, and the United States. Wire-wound guns had been built in various countries since 1890, using wire with a strength of 225,000 lb per sq in., long before such wires had been used for prestressed concrete. High strength concrete of 10,000 lb per sq in. for concrete pipes was being made before 1920. Wire-wound wood stave pipes, tanks, and barrels have been used very extensively in the United States and seldom have failed by tension in the steel. Failure usually was due to shrinkage when empty or partly filled, or due to connection of staves with bottom or to frost. Prestressed or conventional concrete tanks are no exception to these effects.

From an experience with several hundred tanks the writer can mention the following reasons for unsatisfactory performance of some early concrete tanks:

1. The earliest tanks and pipes were of a concrete mixture of 1:3:6; later a mixture of 1:2:4 was thought rich enough. Experienced builders used a richer mixture, especially for elevated tanks.

2. No adequate waterproofing was used. European practice regarded three coats of rich mortar linings, each about ⅛-in. thick, as absolutely necessary. Here waterproofing was provided only after it was discovered that the tanks leaked and the face of the tank was disfigured. This gave the waterproofing contractors a chance to charge a high price for their work. A two or three coat membrane lining of high grade asphalt did a good job; but if such a lining were specified in the beginning, a great deal less concrete and steel could have been used, and the resulting saving would have paid for the waterproofing.

3. The joint of walls and bottom was poorly designed, and the joint broke when the tank was filled. This was emphasized in the past.[21,22]

4. There was always a leak where a sliding joint was used. To the writer's knowledge a sliding joint was first used for a concrete standpipe in the city of Fulton, N. Y., in 1913 and proved unsatisfactory. A very large and high prestressed standpipe was built about 10 years ago in Cincinnati, Ohio; it had a sliding joint and leaked badly.

5. No adequate waterstop was used in the joints of tanks when concreting was interrupted.

Note.—This paper by J. M. Crom was published in October, 1950, as *Proceedings-Separate No. 37*. The numbering of footnotes, equations, tables, and illustrations in this Separate is a continuation of the consecutive numbering used in the original paper.

[20] Civ. Engr. and Contr., Evanston, Ill.

[21] Discussion by L. J. Mensch of "The Relation Between Deflections and Stresses in Arch Dams," by F. A. Noetzli, *Transactions*, ASCE, Vol. LXXXV, 1922, p. 309.

[22] Discussion by L. J. Mensch of "Design of Circular Concrete Tanks," by George S. Salter, *Transactions*, ASCE, Vol. 105, 1940, p. 525.

6. The walls of tanks must be liberally reinforced vertically. One-half inch bars placed 2 ft on centers will not do. As the author showed, there are serious moments acting in the walls, tending to produce horizontal cracks whether the tank is loose or fixed at the bottom. In addition, cracks are caused by the sun shining on one side of the tank.

7. The tanks were not built of exact circular shape.

The writer quite agrees with the figures in line 1 of Table 2 for a prestressed tank. However, the deduction of 35,000 lb per sq in. for secondary stresses might not be sufficient for reasons (not mentioned by the author) such as one-sided sunshine, large temperature variation, and ice formation in the winter.

The author considers the wall resting loosely on the bottom and assumes a coefficient of friction of one half. Serious cracking can be observed in concrete structures with sliding joints in which no sliding steel plates well covered with graphite were provided. It is nearly impossible to produce an even plane surface with a radius of 45 ft for the tank under discussion. Variations of $\frac{1}{8}$ in. to $\frac{1}{4}$ in. in both directions would be considered excellent work. Hence the friction may be much greater in some spots than in others. The author should explain how the coefficient of friction was arrived at. Assume that the movement during prestressing at the bottom was measured. Then, by a very difficult chain of reasoning we can guess at a formula for this movement containing the unknown modulus of elasticity that may vary from 3,000,000 to 5,000,000 lb per sq in., and then the coefficient will vary in proportion.

Another effect is not mentioned. The concrete wall does not remain vertical but has a slope of about 1 to 1,000 at the bottom that will cause breakage of the inside corner in sliding. In addition, the formulas offered by the Portland Cement Association[15] are valid only for walls of uniform thickness, and as the author's tanks have variable thickness from bottom to top, use of these formulas is of doubtful value. If proper sliding cannot be obtained, then the moments in the tank walls may be three times as great as those shown in Fig. 2(*a*) which may explain the cracks found by the author in some tanks.

The use of 2-in. domes in tanks 100 ft in diameter is to be condemned. While the compression in the concrete under full snow load is only 95 lb per sq in., there are many secondary stresses which will crack the dome in time, such as stresses due to prestressing, sunshine, moisture variation, and deviation of shape from a true sphere.

The choice of the unusual wall thickness of 34.6 in. for a conventional concrete tank 90 ft in diameter and 21.4 ft deep was not wise. A great part of the water pressure will be taken up by the cantilever action of the thick wall and transferred to the base slab. This base slab will certainly crack even if it is made four times as thick as the paper recommends. It is assumed further that all circular reinforcing steel be placed near the outside of the tank wall, but this plan has two serious disadvantages:

[15] "Circular Concrete Tanks Without Prestressing," *Bulletin No. ST-57*, Structural Bureau, Portland Cement Assn., Chicago, Ill., October, 1947, p. 3.

a. The shrinkage stresses are doubled; and even without shrinkage stresses the tank will fail at 33% less water pressure than if the horizontal rings are placed near both sides of the wall.[23]

b. Successful practice shows that such wall thicknesses are not necessary. In 1918 a 3,000,000 gal tank was designed for the city of Lansing, Mich. It was 152 ft in diameter and 21 ft high, and according to the author such a tank should have had a wall thickness of 58 in. Actually, it was made 24 in. thick at the bottom and 12 in. thick at the top and had a well-designed connection with a thickened base slab. It never leaked in its 33 years of existence. Another tank, designed in 1910 by the same firm for the town of Norway, Mich., was 35 ft in diameter and 40 ft deep. According to the author the wall should have been 25 in. thick. It was actually 12 in. thick and was so satisfactory that the city built another tank on the identical plans a few years ago. This tank was subjected to unusually severe trials. Sub-zero temperatures of 20° to 40° are common in that part of the country, and every year an ice crust was formed 5 ft thick at the walls and top. Only the fact that the concrete was 1:1:2 mixture saved the tank.

The author claims that ordinary standard reinforcement cracks the concrete when reinforcing is stressed to 20,000 lb per sq in. On this basis all rectangular tanks ought to crack and give trouble. Very long tanks crack on account of shrinkage but can be easily repaired. The writer has built many rectangular tanks and has not found that they leaked because of flexural stress.

The writer wishes to point out that in Table 2, Col. 15, the steel stress due to a shrinkage of 0.0006 should be obtained by the formula $\frac{C\,E_s}{1 + 4\,p\,n}$ as this is a very slow action and E_c becomes $\frac{1}{4}$ or less of the initial modulus. Good practice requires that not only the tank bottom but also the tank walls be kept wet for a long time, thus resulting in a much smaller coefficient of shrinkage. There is no question that prestressing saves a great deal of steel in tanks, high pressure pipes, and culverts, when properly designed.

HERBERT A. SAWYER, JR.,[24] Assoc. M. ASCE.—The experimental evidence presented in this paper for the behavior of high-strength steel reinforcement is an extrapolation of the data presented in 1946 by Howard R. Staley, Assoc. M. ASCE, and Dean Peabody, Jr.,[25] M. ASCE, involving much lower strength steels. An analytical extrapolation seems to indicate that the loss in stress for high-strength steel is significantly higher than shown in Table 1 especially for case (*a*), High Load. The total stress loss from all causes is important in prestressed tank design, as the author clearly demonstrates, so it will be worthwhile to consider these losses carefully.

The following six factors should be considered in the extrapolation of the Staley-Peabody data to obtain the stress loss in high-strength reinforcement: (1) The plastic flow and stress in concrete; (2) the elastic strain of concrete;

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[23] *Bericht No. 27*, Laboratoire Federal d'Essai, Zurich, Switzerland, 1927, p. 8.

[24] Associate Prof. of Civ. Eng., Univ. of Connecticut, Storrs, Conn.

[25] "Shrinkage and Plastic Flow of Pre-Stressed Concrete," by Howard R. Staley and Dean Peabody, Jr., *Proceedings*, American Concrete Institute, Vol. 42, 1946, pp. 229–244.

(3) the time effect; (4) the winding effect; (5) the modulus of elasticity of the steel; and (6) the creep of the wire.

(1) *Plastic Flow and Stress in Concrete.*—Since the initial concrete stress is better maintined by the high-strength steel than it is by the low-strength steel, plastic flow of the concrete with high-strength steel will be greater than with low-strength steel. J. R. Shank[26] has shown that the rate of plastic flow is approximately proportional to the stress for stresses of up to 50% of the ultimate, and the Staley-Peabody tests fall in this range. For this discussion the plastic flow is assumed to be proportional to the average of the initial and final stress in a member. This assumption is accurate enough for present purposes. A more accurate solution, for which there is not sufficient data, would involve the solution of a very complex expression with time, plastic flow, shrinkage, stress, and creep in the steel as variables.

(2) *Elastic Strain of Concrete.*—Concrete which is gradually losing compressive stress expands elastically, and this expansional strain component partly cancels out the contractive, stress-reducing, strain components from plastic flow and shrinkage. This cancelling-out effect is smaller with high-strength steel because less concrete stress is lost with high-strength steel.

(3) *Time Effect.*—Loss of stress from shrinkage and plastic flow should be based on a period much longer than the 400-day losses of the Staley-Peabody tests. According to tests at the University of California,[27] in Berkeley, roughly 80% of the total deformation from plastic flow occurred in 1 year for specimens under constant load. Of course, a correction must be made for the decrease in load on prestressed concrete. Also, according to tests reported by Raymond E. Davis,[28] roughly 75% of the shrinkage in 10 years takes place in the first year. This percentage, based on tests performed about 1910, seems low. In computations that follow, the shrinkage at 400 days is arbitrarily assumed to be five sixths of the shrinkage at 25 years. Of course, changing moisture conditions would change these figures. The values given have been estimated for conditions similar to the Staley-Peabody test conditions, which, in terms of humidity and loading, are severe and conservative and correspond to those pertaining to an unfilled tank. If the tank were always full, the concrete stress, and hence the plastic flow, would be insignificant, and the high humidity on one side of the tank wall would retard shrinkage. Obviously few, if any, tanks could be designed for this assumption. Incidently it follows that the water-tightness of prestressed tanks under all conditions cannot be judged by the behavior of tanks that have very rarely been less than two thirds full. Keep them busy, and they stay out of trouble.

The tests of Messrs. Staley and Peabody were conducted in the Department of Building, Engineering, and Construction at Massachusetts Institute of Technology, Cambridge, Mass. This department has continued observation of the specimens of these tests and has kindly made available the more recent data

[26] "The Mechanics of Plastic Flow of Concrete," by J. R. Shank, *Proceedings*, American Concrete Institute, Vol. 32, 1936, pp. 149–180.

[27] "Plastic Flow and Volume Changes of Concrete," by Raymond E. Davis, Harmer E. Davis, and Elwood H. Brown, *Proceedings*, American Society for Testing Materials, Vol. 37, Pt. II, 1937, pp. 319–320.

[28] "Volume Changes in Concrete," by Raymond E. Davis, *Proceedings*, American Concrete Institute, Vol. 26, 1930, p. 429.

on these specimens. The data are of some help in estimating, for wire-reinforced concrete, the time effect of shrinkage and plastic flow subsequent to a 400-day age. Data of the unstressed control specimens indicate that the shrinkage at 400 days is about nine tenths of the shrinkage at 1,800 days for both gunite and concrete. The forgoing assumption as to shrinkage at 25 years seems to agree with these results. Data of the specimens prestressed with rods indicate that, for final stresses of from 25% to 34% of the initial stresses, the calculated plastic flow at 400 days averages about 95% of the calculated plastic flow at 1,800 days. This finding probably does not disagree with the foregoing assumption as to total plastic flow with a final stress maintained (with wire reinforcement) at a much higher percentage of initial stress.

(4) *Winding Effect.*—Of course it is the winding of the wires in tension around the initially unstressed concrete shell which gradually compresses the shell. On the average, when and where the wire makes contact with the shell,

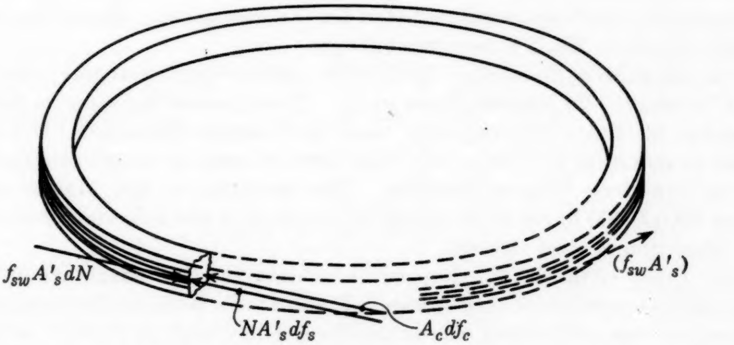

FIG. 5.—INCREMENTS OF FORCE FROM WINDING

only half this compression exists in the shell since the tensioned wires have been applied to only one side of the point of contact. Further compression tends to unload the wire. The approximate ratio to which the initial steel tension must be increased to cancel this loss is

$$\frac{f_{sw}}{f_{sa}} = \frac{f_{sa} + \frac{fc}{2} n_w}{f_{sa}} = 1 + \frac{p_w n_w}{2} \qquad (2)$$

in which f_{sw} is the steel stress in winding; f_{sa} is the average steel stress immediately after winding; p_w is the ratio of steel area to the concrete area immediately after winding; and n_w is the ratio E_s/E_c at the time of winding.

The more exact solution for this loss should probably be included here, if for no other reason than to prove that Eq. 2 is on the safe side and is accurate enough for all except extremely high values of $p\, n$. Consider an initially unstressed thin ring of cross-sectional area A_c, about which many turns of wire (each turn of cross-sectional area A'_s and winding unit tension f_{sw}) are being wound (Fig. 5). Increments of tension introduced by the wire being wound must be bal-

anced (by statics) by equal increments of compression in the ring and previously wound wire:

$$f_{sw} A'_s \, dN = A_c \, df_c + N A'_s \, df_s \quad \text{(3)}$$

in which N, f_c, and f_s are variables, N being the number of turns of wire at any time during the winding process. Also, if the materials are assumed elastic during the winding process

$$df_s = n_w \, df_c \quad \text{(4)}$$

Solving these two differential equations simultaneously, the ratio of Eq. 2 becomes:

$$\frac{f_{sw}}{f_{sa}} = \frac{p_w n_w}{\log_e (1 + p_w n_w)} \quad \text{(5)}$$

A 10% error would not be out of line with the accuracy of the assumptions underlying this analysis, and it is only for $p_w n_w > 1$ that the error of the stress increment of Eq. 2 compared to that of Eq. 5 exceeds 10%. Hence, for practical design purposes, Eq. 2 is accurate enough.

(5) *Modulus of Elasticity of Steel.*—The author states that this value for the wire "averages" 26,000,000 lb per sq in. This ls about the same as the secant modulus for the wire commonly used by Gustave Magnel.[29] The ratio of stress to strain for a decrease of a high stress in steel is more accurately given by the low-stress tangent modulus. This modulus, as Mr. Magnel shows, is about 29,000,000 lb per sq in. which will be used in the following computations for relaxation of steel stresses.

(6) *Creep of the Wire.*—Experimental data for this phenomenon at non-elevated temperature is very sparse. The author's value of a maximum of 5% stress-loss from this effect was evidently not included in Table 1 or in other calculations of the paper. The effect of creep may be neglected for the low stresses in low-carbon steel of the Staley-Peabody tests but not for high stresses in high-strength steel, especially for wire that receives tension by cold drawing at the time of winding. Mr. Magnel found strains in Belgian wire from creep of from 1% to 16% of the initial strains. This variation depended, primarily, on the rate of application of the initial stress, magnitude of initial stress relative to strength of wire, type of steel, and condition of restraint. In a constant-length test of the wire (mentioned herein under factor (5)) stressed in 2.5 min to 123,000 lb per sq in. (57% of its ultimate tensile strength), the loss in stress was 12% in 14 days. An arbibrary loss of 8% of the initial stress after winding has been assumed for cold-drawn wire initially stressed to about 75% of its ultimate in the computations that follow. There are many reasons why this may or may not be conservative enough, but, even if more data on creep were on hand, an accurate quantitative estimate of it would be almost impossible because of the interaction of creep strains with all the factors that cause strains in the concrete. Further investigations of this potentially important factor are very much needed. Perhaps the author can provide some more information on creep from his own experience in his closure.

[29] "Creep of Steel and Concrete in Relation to Prestressed Concrete," by Gustave Magnel, *Proceedings*, American Concrete Institute, Vol. 44, 1948, pp. 485–500.

The loss of stress in the steel wire has been computed in Table 4 for the same cases considered by the author in his Table 1. In this table, the six factors, (1) to (6), have been taken into account. The claim of great accuracy cannot be made for these computations. It is not possible for such computations to be very accurate, and for this reason only two significant figures are carried. This writer believes, however, that the inclusion of each of these six factors has

TABLE 4.—REVISED STUDY OF STRESS LOSSES

Item	Description[a]	Reference	High Load		Low Load	
			Gunite	Concrete	Gunite	Concrete
(1)	(2)	(3)	(4)	(5)	(6)	(7)
a	Average E_c at the time of winding	238[b]	3.34×10^6	3.30×10^6	3.34×10^6	3.30×10^6
	Winding stress f_{ws}—					
b	for f_{ws} = 150,000 lb per sq in.	Factor 4[c]	160,400	160,500	154,000	154,100
c	for 3% error = 1.03b	Crom[d]	165,200	165,300	158,600	158,700
d	Average[e] f_c	236[b]	1,710	1,540	660	610
e	Total concrete strain after reload[e]	236[b]	−0.00086	−0.00112	−0.00081	−0.00096
f	Average[e] E_c	242[b]	3.67×10^6	4.35×10^6	3.67×10^6	4.35×10^6
g	Change in stress[e] f_c	236[b]	1,360	1,710	510	640
h	Elastic strain, concrete[e] = g/f	Factor 2[c]	+0.00037	+0.00039	+0.00014	+0.00015
i	Shrinkage strains after reload[e]	232[b]	−0.00045	−0.00061	−0.00061	−0.00079
j	Plastic flow after reload[e] = e − h − i	....	−0.00078	−0.00090	−0.00034	−0.00032
k	Initial f_c (second reload)	236[b]	2,390	2,390	920	935
l	Assumed loss in f_c-high tensile wire; see item y	....	1,000	1,200	320	370
m	Assumed average f_c, high tensile wire = k − l/2	....	1,900	1,800	760	750
n	Plastic flow, high tensile wire, 400 days = m k/d	Factor 1[c]	−0.00087	−0.00105	−0.00039	−0.00039
o	Plastic flow, at 25 yr = $n\left(1 + 0.25\,\frac{k - l}{m}\right)$	Factor 3[c]	−0.00103	−0.00122	−0.00047	−0.00046
p	Elastic strain, concrete = l/f	Factor 2[c]	+0.00027	+0.00028	+0.00009	+0.00009
q	Shrinkage strain 6 to 400 days	232[b]	−0.00071	−0.00093	−0.00071	−0.00093
r	Shrinkage strain at 25 yr = 1.2q	Factor 3[c]	−0.00085	−0.00112	−0.00085	−0.00112
s	Total strain = o + p + r	....	−0.00161	−0.00206	−0.00123	−0.00149
t	Partial loss of stress in steel = $E_s s$	Factor 5[c]	46,600	59,800	35,700	43,200
u	Loss in steel from creep = 0.08 × 150,000	Factor 6[c]	12,000	12,000	12,000	12,000
v	Low humidity loss	Crom[d]	3,000	3,000	3,000	3,000
w	Maximum total loss of steel stress = c − 150,000 + t + u + v	....	76,800	90,100	59,300	66,900
x	Maximum percentage loss in f_{ws} = 100 w/c	....	46.5	54.5	37.5	42
y	Loss of stress in concrete = $\frac{t + u + v}{150{,}000}\,k$	....	980	1,190	310	360
z	Percentage loss, concrete stress = 100 y/k	....	41	50	34	39

[a] In col. 2, stresses f and moduli E are pounds per square inch; strains and plastic flow are inches per inch. The nonitalic letters a, b, c, . . ., z denote the numerical values recorded opposite the corresponding letters in Col. 1. [b] "Shrinkage and Plastic Flow of Pre-Stressed Concrete," by Howard R. Staley and Dean Peabody, Jr., *Proceedings*, American Concrete Institute, Vol. 42, 1946, at the page cited in Col. 3. [c] Factors 1 to 6, defined in this discussion. [d] As given in the paper under discussion. [e] Computations necessary to evaluate the approximate plastic flow of the concrete for the Staley-Peabody tests are listed in lines d to j, inclusive.

greatly improved the accuracy of the result. Also, even if the maximum error per factor were ± 25%, the probable error for the summation would be ± 10% to 15%.

The percentage of stress lost by the steel, item x of Table 4, is not equal to the percentage of stress lost by the concrete, item z, because the steel loses some stress before the concrete has reached its maximum initial stress. Items d through j of Table 4 outline the computations necessary to evaluate the approximate plastic flow of the concrete for the Staley-Peabody tests. The most important conclusions to be drawn from Table 4 are:

(*a*) The stress loss from shrinkage and plastic flow is not a constant, as indicated in Table 1, but a variable depending mainly on the wire-winding stress and to a lesser extent on the factor $p\ n$.

(*b*) This variation is so important that actual stress losses for high-tension reinforcement are two or three times the losses indicated in Table 1, assuming full concrete shrinkage but neglecting the 3% tension-error loss and the low humidity loss. Stress-loss allowances for high-carbon wire should be greater than for rods, not less as indicated on Table 2.

(c) Discrepancies of this magnitude indicate the need for careful reconsideration of stress losses for other types of prestressed concrete construction.

On the other hand, several of the most important stress-reducing factors could be partly eliminated if special measures were taken:

1. The loss from shrinkage of the concrete would be reduced if the winding operation were delayed until much of the shrinkage had taken place. For Staley-Peabody shrinkage conditions, winding 21 days instead of 6 days after placement would reduce stress loss by about 10,000 lb per sq in. Such a delay could possibly disrupt the construction schedule, making it economically unfeasible for the average single-tank job.

2. Mr. Magnel's experiments[29] indicate that special steel wires (perhaps heat-treated after drawing) may be developed for which the magnitude of creep is almost negligible—another potential 10,000 lb per sq in. reduction in the stress loss of Table 4. Whether a stress-saving of this magnitude would be sufficient to balance the extra cost of such a wire is a question the writer cannot answer.

3. As stated before, stress losses from both shrinkage and, to a lesser extent, from plastic flow, would be greatly reduced from those of Table 4 if the tank were filled to near-capacity most of the time.

The author did not specify any of these special measures for the prestressed-wire design of Table 2; thus this design roughly corresponds to the low-load condition of Tables 1 and 4, the winding stress being slightly lower and $p\ n$ being roughly twice as high. If the assumed stress loss of this design were to be changed from 35,000 lb per sq in. to 60,000 lb per sq in. to correspond with item w, Table 4, the final stress in the concrete for full tank, Col. 20, Table 2, would be 100 lb per sq in. in tension instead of 78 lb per sq in. in compression. The change in steel stress in case of cracks, Col. 22, Table 2, would be 14,000 lb per sq in., and the maximum increase in the circumference of tank, Col. 23, Table 2, would be 1.82 in.

How serious are these changes in stresses for this design? The usual engineering design guards against a failure that would involve loss of life, wealth, and the structure itself. Thus, of necessity, the usual engineering design is extremely conservative. This conservativeness is still satisfied by the storage tank in Table 2 because even with the concrete cracked the stress in the wire is only about 100,000 lb per sq in., whereas its minimum ultimate is 200,000 lb per sq in.

Prestressed tank design is different from the usual design in that it also involves a calculated guarding against cracking failure, a failure that, except for the corrosion danger, involves only leakage losses and unsightly stains. Traditional engineering conservativeness is certainly uncalled for here, especially when this leakage would occur only on the ocacasional tank, which would be empty part of the time. (Corrosion would remain a problem, especially since the wire is so small and since the tensioning method used for prestressed tanks precludes the use of galvanized wire.)

Therefore, in the writer's opinion, these changes in stresses are not serious, except for corrosion, and the prestressed wire design is still superior to the prestressed rod design in saving materials.

The revisions involved in Table 4 would be of more vital importance for tanks with higher hoop tensions, for which there would be a tendency to use higher quality materials and higher working stresses.

As the author states, an overload is extremely unlikely, but it should be mentioned that the crack resistance of a wired tank to overload is much inferior to that of a rodded tank. The ultimate tensile resistance of the concrete would be approached much more rapidly in the wired tank with an increasing overload. After this tensile resistance had been exceeded, cracks would open up about eight times as fast as those in a prestressed rod tank (eight being the ratio of the steel areas). This might be an advantage; the overload would soon leak out of a wired tank! Of course, with removal of the overload, the cracks would disappear, a phenomenon typical of prestressed concrete.

Computations similar to those of Table 4 could be used to determine approximate stress losses for any prestressed tank or pipe design. It would be possible also to develop an algebraic expression that would give an approximate stress loss for any design almost automatically.

J. M. Crom[30].—The discussion by Mr. Mensch was most interesting since he is one of the pioneers in concrete tank construction and his account of tanks built during the past are valuable. His comments relative to sliding joints are of particular interest since any uneven settlement of the tank foundation will allow leakage if the wall and foundation are not doweled together or if there is not some kind of flexible key connection between them. However, experiences with more than 200 tanks having sliding joints indicate that leakage develops only occasionally and that it usually can be sealed at small expense. The writer has on several occasions made careful studies of the Fulton, N. Y., tank mentioned by Mr. Mensch, before it was demolished as a failure. As would be expected, the base of the wall moved outward. However, when the tank was in the empty condition, the walls did not return to the original position, perhaps partially because of friction, but chiefly on account of the chemical deposits that had formed in the vertical cracks, thus increasing the circumference and in effect simulating the conditions of a prestressed body. The percolation of the water through the walls caused rusting of the band rods, and the expansive force of the rust and the freezing of the water caused the

[30] Vice-Pres., The Preload Enterprises, Inc., New York, N. Y.

spalling of large areas of the concrete outside of the reinforcement. The design of this tank is described and illustrated elsewhere.[31]

Regarding the resistance to movement between wall and foundation slab because of friction, an accurate satisfactory coefficient that would apply to all jobs cannot be fixed. However, experience with tanks constructed has resulted in some data that are of help. For instance, during the wire winding operations when the first movement of the wall occurs, notes are kept showing the number of bands in place. These data, together with a knowledge of the condition of the joint prior to pouring, indicate a figure that can be used by the designer. In some instances this coefficient has been low, indicating that with proper care it can be kept within practicable limits.

Designs are under consideration that include the tying of the floor to the sidewalls by means of the vertical prestressing units after the specified amount of inward movement of the wall has occurred. Experiments having to do with heavy rubber dumbbell waterstops that extend from the wall through the joint and into the foundation slab have also been undertaken.

Mr. Mensch discusses a tank built for the City of Lansing, Mich., during the year 1918, and also another built in 1910 for the Town of Norway, Mich. It would be interesting to know whether or not these tanks were ever exposed to severe drying conditions such as would occur during a long, hot, dry period while the tank was empty and whether or not the tanks were covered partially or wholly with earth. If the walls were continuously damp it is doubtful that the shrinkage exceeded a coefficient of 0.0003, in which case the unit tensile band stresses in the concrete of the walls never exceeded 400 lb per sq in., which should be within the strength of the type of concrete used. The band steel would still have been in compression and thus impose an extra tensile load on the concrete.

The thickness to specify for dome shells is largely a matter of conscience. There are scores of tanks now in regular use that have shell domes 2 in. in thickness. Some of these shells show cracking, but this is also the case with domes of much greater thicknesses. All are structurally strong if the form work over which they are cast is true. The meridional and circumferential stresses are under 200 lb per sq in.

Sometime prior to 1920 an 83-ft diameter shell was built over an oil tank in California. This shell was still giving good service when it was inspected by the writer during 1948. It had a thickness of 1.5 at the crown and about 2 in. at the ring. It contained a number of cracks that seemed to be causing no damage.

Concerning a reduced value of the factor E_c after a lapse of time, as pointed out in the last paragraph of Mr. Mensch's discussion, it is not believed that this condition will occur in all cases in which the shrinkage coefficient has reached a value of $C = 0.0006$. In several instances within the experience of the writer this value was reached very soon after the completion of the construction work.

[31] "Hool and Johnson," *Concrete Engineers Handbook*, McGraw-Hill Book Co., Inc., New York, N. Y., 1918, p. 769.

Mr. Sawyer discusses the losses of stress caused by the creep of the wire under high stress. There is no question but that the creep is considerable when the stresses are very high. Luckily, however, a large percentage of it takes place during the first periods of the stressing, and since the initial stresses are not unduly high, it is believed, and it is reasonably well verified, that the losses because of this fact do not exceed 3%.

The results of tests made by the writer showed that with a constant length, and at the stresses used in tank work, the total creep over a period of several months did not exceed 6%. Since about $\frac{2}{3}$ of this creep took place during the first, second, and third minutes of the stressed period, a similar phenomenon could be expected in the wire-winding operation. For example, in a winding operation the pull on the wire is constant as it comes from the machine. Since some of the creep takes place almost instantly, the release of slack is taken up over a length of wire to the point at which the friction between wall and wire prevents further take-up. Also, the wire is somewhat increased in temperature by the stressing mechanism. The return of the wire to normal temperature fully takes up any loss incurred by creep.

Many critics have deprecated the assumptions of stress losses of the magnitude of 40,000 lb per sq in., and since Mr. Sawyer would have them at 60,000 lb per sq in. in the case of the usual method of prestressing, there is comfort in his statements. He also supplies further data in connection with the Staley-Peabody tests as well as the development of formulas to simplify the computations involved in the designs of prestressed structures.

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